VINCENT

Investigation of a Steel Highway Bridge

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INVESTIGATION OF A STEEL HIGHWAY BRIDGE

BY

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THESIS

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I recommend that the thesis prepared under my supervision by CHESTER ANDRUS VINCENT entitled Investigation of a Steel Highway Bridge be approved as fulfilling this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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Recommendation approved

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INVESTIGATION OF A STEEL HIGHWAY BRIDGE.

INTRODUCTION.

At the present time there are a great number of bridges throughout the country designed and built from twenty to thirty years ago, when steel bridge engineering was in its infancy. The use of iron was gradually giving way to that of steel in bridge building, as the properties of steel became more and more understood. At that time a great difference of opinions existed concerning the methods of computing live load stresses and the amounts of impact, while at present our standard specifications vary but little. On some structures dead load has been added, such as pavements, etc. It is evident, then, that these bridges must be carefully investigated with regard to present conditions, and their efficiency under present maximum loadings determined.

The purpose of this thesis is the investigation of the Gilbert Street highway bridge crossing the Vermillion River at Danville, Ill. This bridge was built of steel in 1893. Some time later it was used by the Illinois Traction System, but when certain details were found to be failing, its use by the Traction Co. was discontinued. The bridge has since been paved with wooden blocks, and is now used exclusively for highway traffic.

The structure is a deck bridge approximately 1100 feet long, consisting of two spans of 280 ft. each, two smaller spans of 80 ft., four of 60 ft. and five towers.

All the data used in the computations were obtained from measurements of the bridge.

The investigation will be taken up in the following general order:

PART I.

FLOOR SYSTEM.

1. Floor
2. Stringers
3. Floor Beams

PART II.

INVESTIGATION OF MAIN TRUSSES.

1. Upper Chords
2. Lower Chords
3. End Posts
4. Vertical Web Members
5. Diagonal Web Members
6. Details

PART III.

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(same as Part II.)

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PART VI.

PHYSICAL CONDITIONS.

1. Corrosion
2. Paint, etc.

CONCLUSIONS.

Ostrop's Standard Specifications for Highway Bridges will be used throughout. As this bridge is subject to medium city traffic, Ostrop's Class (b) loadings will be used in the computations of live load stresses. These loadings are as follows: For the floor system and local truss members, a concentrated load of 30,000 lbs., distributed on two axles 8 ft. centers and 5 ft. gauge (occupying a length of 20 ft. and a width of 10 ft.), and upon the remaining area of the floor, including sidewalks, a load of 90 lbs. per sq. ft. For the trusses or girders, 90 lbs. per sq. ft. of entire roadway and sidewalks for spans of 100 ft. or less, 70 lbs. per sq. ft. for spans of 200 ft. or over, and proportionally for intermediate spans.

Impact to be computed from the following formula:

$$I = S \frac{100}{300 + L}$$

where S equals maximum live load shears, moments, or stress and L equals the loaded length of the span in feet.

PART I.

INVESTIGATION OF FLOOR SYSTEM.

Art. 1. Floor

The floor is made up of four-inch wooden block paving placed on three-inch plank flooring. As the exact action of paving and flooring under load is not known, the assumptions will be made that the concentrated wheel load is distributed over two blocks or a strip 8 inches wide, and that the paving does not arch but merely transmits the load to the flooring. The floor planks are continuous beams, but for the sake of simplicity will be considered as simple beams, which is also on the side of safety. From the specifications, a concentrated load of 30,000 lbs. on four wheels gives 7500 lbs. on each wheel. As the stringers are spaced 26 inches center to center the bending moment in the flooring is 3750 x 9 equals 33,700 inch pounds. For rectangular beams,

$$S = \frac{6\ M}{bd^2}$$

which gives a unit stress of 1890 lbs. per sq. in. The allowable stress is 1300 lbs. per sq. in. and the efficiency 69.0%. It will be necessary to consider only horizontal shear, as wood has a low shearing resistance with the grain.

$$H = \frac{3}{2}\,\frac{V}{bd}$$

V is equal to 3750 and H equals 156 lbs. per sq. in. The allowable stress is 160 lbs. per sq. in. and the efficiency 102.5%.

Art. 2. Stringers.

The floor stringers are $5\frac{1}{2}$" x $11\frac{1}{2}$" timbers spaced 26 ins. center to center with a 20-ft. span. Assuming the total load of one wheel is taken by two stringers, this would give two loads of 3750 lbs. 8 ft. apart on one stringer. When these are in position to give maximum moment, two feet of the uniform live load will also act on one end of the stringer. The dead load including the weight of the stringer is 60 lbs. per lin. ft. The maximum moments are: Live load 288,000, impact 90,000, and dead load 34,000 inch pounds, making a total of 412,400 in. lbs. This gives an extreme fiber stress of 3410 lbs. per sq. in. from the formula given in Art.1. The allowable stress is 1300 lbs. per sq. in.,and the efficiency of the stringer in bending is 38.2%.

The maximum shear is 6000 lbs. live load, 1870 lbs. impact, and 600 lbs. dead load, or a total of 8470 lbs. From the formula given in Art. 1. the horizontal shear is 201 lbs. per sq. in. The efficiency is 79.7%.

The sidewalk stringers are 14" x $3\frac{1}{2}$" beams spaced 2 ft. 6 ins. on a 20-ft. span and carry a live load of 225 lbs. and a dead load of 34 lbs. per ft.,which gives a moment of 197,000 in. lbs., and an extreme fiber stress of 1725 lbs. per sq. in. The bending efficiency is 75.5% The shear is 3290 lbs. and H equals 100 lbs. per sq. in.,and the efficiency is 160%.

Art. 3. Floor Beams.

The floor beams are 15-inch 55-lb. I's 33 ft. long, with supports 20 ft. apart, leaving two cantilever arms of 6 ft. 6 inches. The roadway is 23 ft. wide with a 5-ft. sidewalk on each side. The dead load on the roadway is 600 lbs.,

and on the sidewalks 230 lbs. per ft. of beam. The live load on the sidewalk is 90 x 20 = 1800 lbs. per ft. The maximum concentrated load is 30,000 x 16 divided by 20 equals 24,000 lbs. on two wheels 5 ft. apart, or 12,000 lbs. each. The roller occupies a space of 10 by 20 ft., and outside of this is the uniform live load of 90 lbs. per sq. ft. The two outside strips give a load of 1800 lbs. per ft., and the load due to the remaining area in front and behind the roller is 520 lbs. per ft.

There are two conditions to be investigated for maximum moment. Dead and live load on the cantilever producing bending at the support, or dead load only on the cantilever and full live and dead load on the roadway. The total load on the cantilever arm including impact is 2560 lbs. per ft., or a bending moment of 648,000 inch pounds. For the second condition the dead load on the cantilever produces negative moment, and the maximum positive moment will be under the wheel nearest the center of the span. The wheel producing the maximum is at a distance from one support equal to the distance of the center of gravity of the two wheel loads from the other. This position of the loads will give a total reaction on the support nearest the wheel considered, of 37,390 lbs. and a moment of 2,535,000 inch pounds. The unit stress is the moment divided by the section modulus, 68.1, or 37,200 lbs. per sq. in. The efficiency is 16,000 (the allowable stress) divided by 37,200, or 43.0%.

The maximum reaction for shear is 57,200 lbs., which divided by 16.18, the area of the section, gives a unit stress of 3530 lbs. per sq. in., and an efficiency of 284.0%.

Fig. 1.

Fig. 2.

Fig. 3.

SKETCH OF MAIN TRUSS.

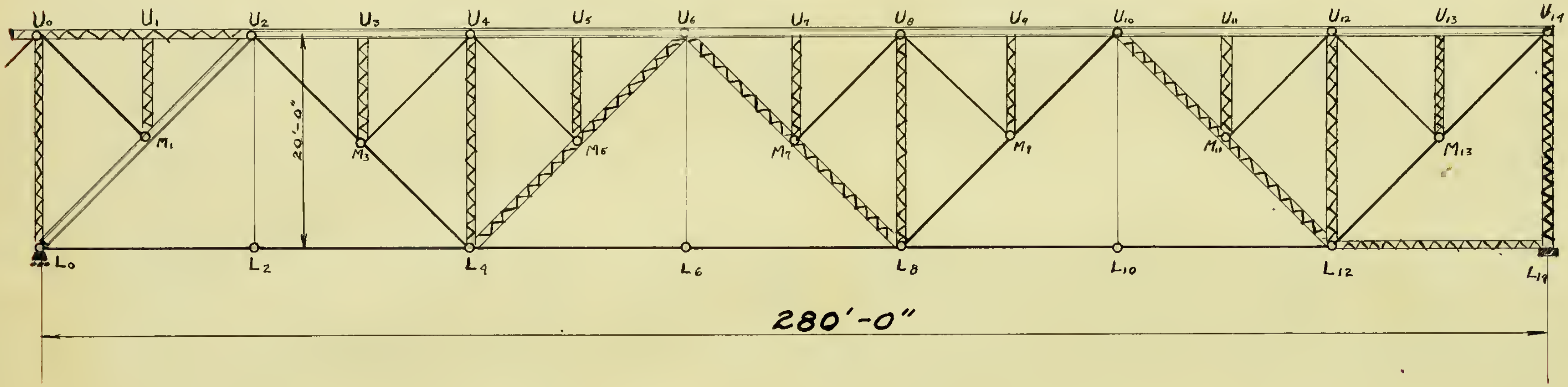


Fig. 4.

TABLE I.

WEIGHT OF MAIN SPAN.

No.	Member	Length in ft,	Wt. per ft. pounds	Wt. member	Total Wt.
2	End Post Details	56.6	114.0 25%	6450 1610 8060	16,120
6	Posts Details	40.0	34.0 25%	1360 340 1700	10,200
14	Posts Details	20.0	19.5 25%	390 100 490	6,860
6	Diagonals Details	56.6	51.2 25%	2900 720 3620	21,700
4	Chords Details	80.0	95.2 25%	7620 1900 9520	38,080
4	Chords Details	40.0	72.2 25%	2888 742 3630	14,520
32	5" x1$\frac{1}{4}$"Eyes	33.6	21.25	714	22,820
8	5"x2 " "	33.6	36.13	1212	9,720
8	5"x 2" "	33.6	34.0	1140	9,130
8	5"x " "	33.6	13.81	464	3,710
28	1" bars	33.0	3.4	112	3,140
4	Chords Details	40.0	16.4 25%	655 165 820	3,280
2	Posts Details	40.0	60.4 25%	2416 604 3120	6,240
16	Struts	20.0	25.0	500	8,000
	Laterals Details Pins, etc.				6,180 2,000
				TOTAL	184,700

TABLE II.

MAIN SPAN STRESSES.

Member	Dead Load	Live Load	Impact	Total
L_0L_4	95,600	138,600	23,900	258,100
L_4L_8	196,200	277,200	47,800	521,200
L_8L_{12}	164,100	231,000	39,800	434,900
U_0U_2	-6,100	-11,600	-3,400	-21,100
U_2U_6	-173,000	-242,600	-41,800	-457,400
U_6U_{10}	-206,800	-288,800	-49,900	-545,500
$U_{10}U_{14}$	-108,800	-150,200	-25,900	-284,900
L_0U_0	-12,200	-23,100	-6,800	-42,100
L_4U_4	-24,400	-46,200	-11,000	-81,600
$L_{14}U_{14}$	-115,100	-161,700	-27,900	-304,700*
M_1U_1	-12,200	-23,100	-6,800	-42,100
L_0M_1	-135,500	-195,000	-33,600	-364,100
M_1U_2	-126,800	-178,600	-28,800	-334,200
U_2M_3	108,600	156,400	32,800	297,800
M_3L_4	100,800	140,000	28,000	268,800
L_4M_5	-41,700	-93,400	-20,300	-155,400
M_5U_6	-33,800	-77,000	-15,500	-126,300
U_6M_7	14,980	-39,600	-8,600	
M_7L_8	7,150	-58,000	-13,400	
L_8M_9	52,000	93,400	20,300	165,700
M_9U_{10}	59,800	109,800	25,100	194,700
$U_{10}M_{11}$	-78,600	-123,600	-23,200	-225,400
$M_{11}L_{12}$	-86,500	-140,000	-28,000	-254,500
$L_{12}M_{13}$	145,700	195,000	33,600	374,300
$M_{13}U_{14}$	153,500	211,400	38,400	403,300

* Wind stress must be considered.

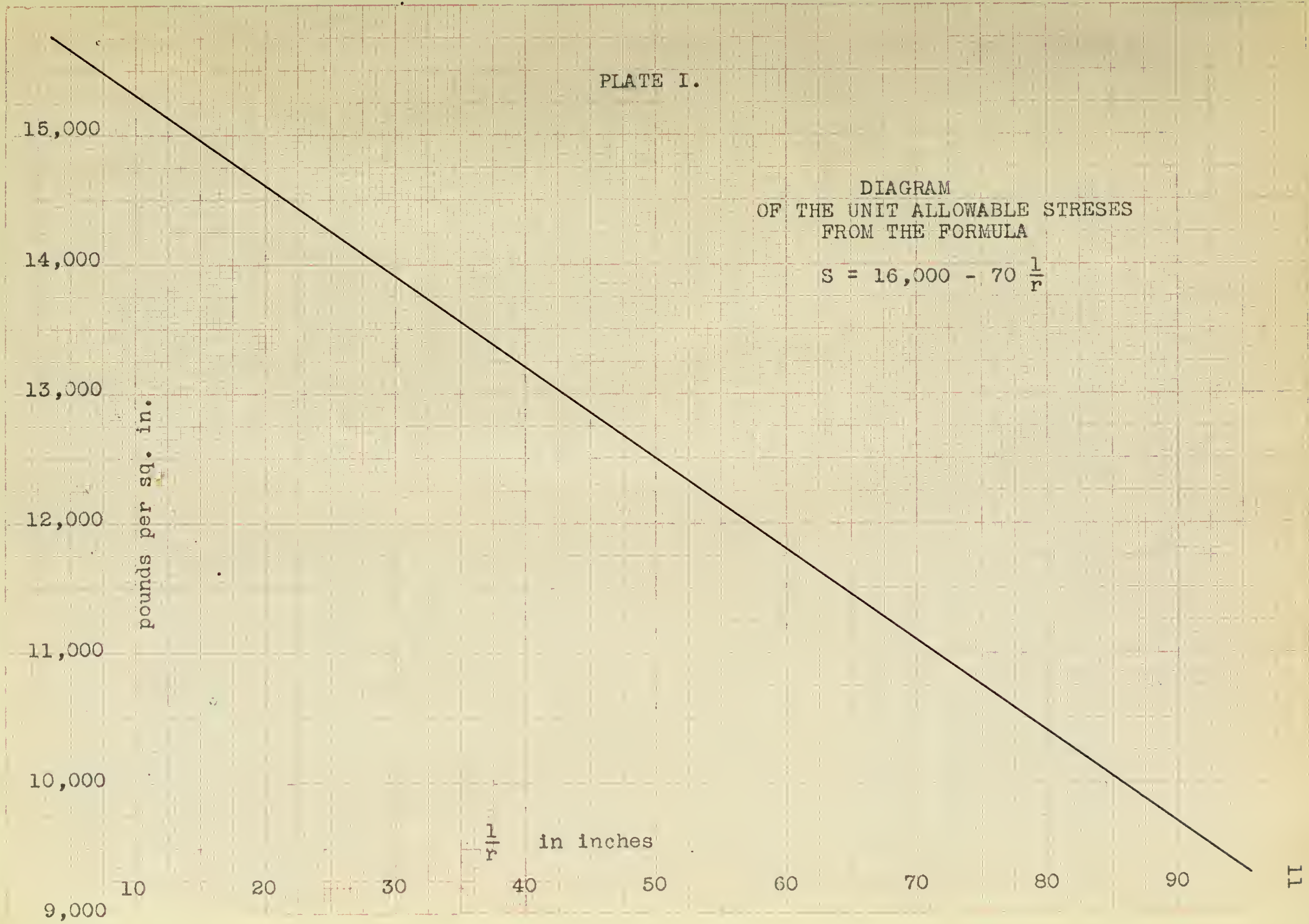


PLATE I.

DIAGRAM
OF THE UNIT ALLOWABLE STRESES
FROM THE FORMULA

$$S = 16{,}000 - 70\,\frac{l}{r}$$

PART II.

INVESTIGATION OF MAIN TRUSSES.

There are two main spans of 280 ft. each, which are similar except for the center pier which is set on a skew of about 63 degrees from the center line of the truss (Fig. 2). The span length is for the longer truss. The trusses are of the deck sub-divided Warren type with fourteen panels 20 ft. each (Fig. 4). The depth is 40 ft. and the width 20 ft. center to center of trusses.

From Table I the weight of the steel of one span including lateral system is 184,700 lbs., or a panel load of 6600 lbs. on each truss. In the computation of stresses one half of this was considered as acting on the lower chord. The weight of one panel of floor and floor beam is 8900 lbs. on each truss. The live panel load is 70 x 20 x 33 equals 46,200 lbs., or 23,100 lbs. on each truss. As the truss is not symetrical about the center (Fig. 4), the stresses were computed for the entire span. The angle of the web members with the vertical is 45 degrees; tan θ = 1 and sec θ = 1.414. The analytical method of computing stresses becomes simple when the sub-divided panels are considered seperately as king post trusses. For the live load stresses in the web members, the truss was loaded for maximum shear at the panel considered. The results were checked by graphics and are tabulated in Table II.

Plate I is the curve for the unit allowable stresses in compression members from the formula:

$$S = 16,000 - 70 \frac{l}{r}$$

Art. 1. Investigation of Upper Chords.

In the upper chords there are three conditions to investigate: direct stress, bending due to eccentricity, and bending due to the weight of the member. From the specifications, if the latter does not exceed 1600 lbs. per sq. in., it may be neglected. As it was found to be under that amount in all cases, it will not be considered. The total unit stress was computed from the formula:

$$S = \frac{P}{A} + \frac{M_e y}{I - \frac{Pl^2}{CE}}$$

P = direct load

A = total area of the section

M_e = moment due to eccentricity in inch pounds

y = distance to extreme fiber in inches

I = moment of inertia of the section

l = unsupported length of the member in inches

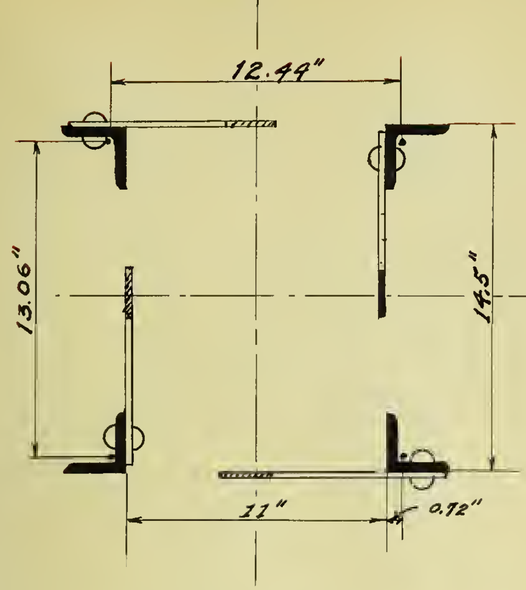
C = constant

E = modulus of elasticity of the material

For U_6U_{10} (Fig. 6), I = 818.8 in.^{4}, e = 1.2 ins., A = 27.98 sq. ins. and the maximum total stress P, Table II, is -545,500 lbs. This gives an extreme fiber stress of 27.070 lbs. per sq. in. and an efficiency of 59.2%, the unit allowable stress being 16,000 lbs. per sq. in. in combined stresses.

U_2 U_6 has the same section as U_6 U_{10} and a stress of -457,400 lbs. The resulting unit stress is 22,550 lbs. per sq. in., and the efficiency 74.4%.

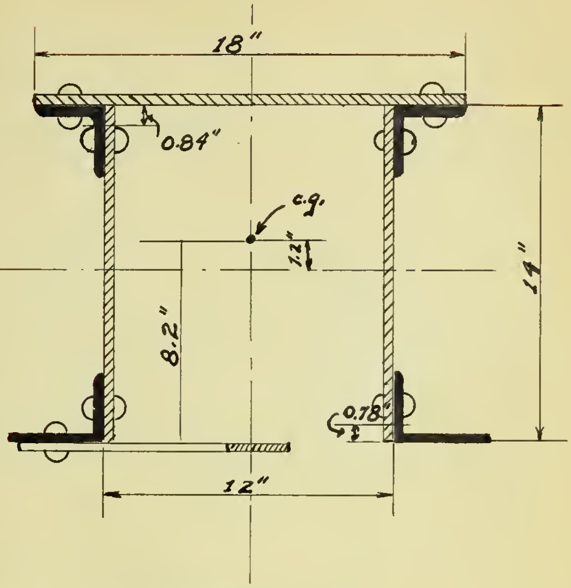
Fig. 5.



SECTION $U_0\ U_2$

4 L's $2\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{1}{4}$"

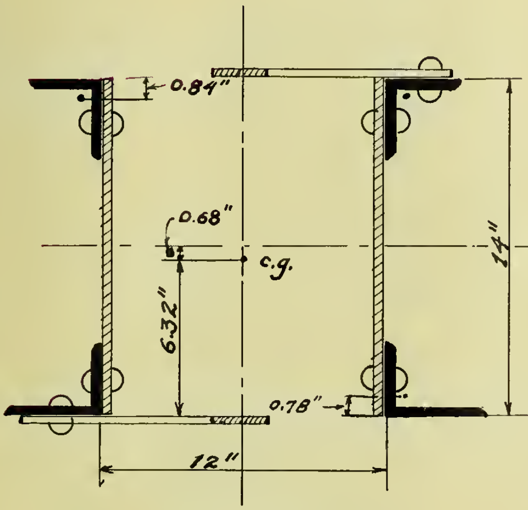
Fig. 6.



SECTION $U_6\ U_{10}$

2 L's 3" x 3" x 3/8"
2 L's 4" x 3" x $\frac{1}{2}$"
2 Pls. 14 " x 3/8"
1 Cov. 18" x 3/8"

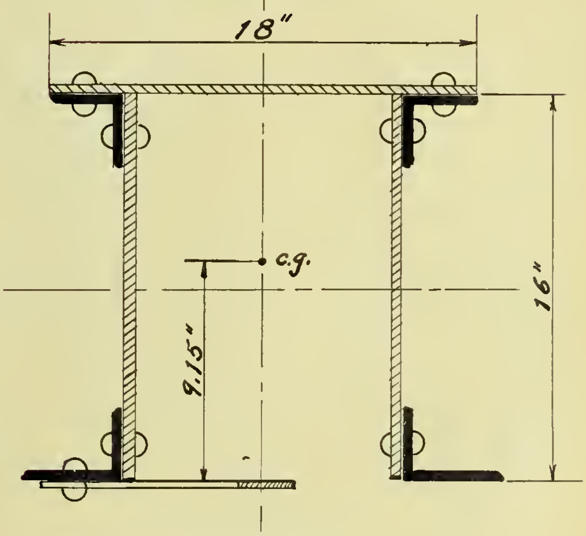
Fig. 7.



SECTION $U_{10}\ U_{14}$

2 L's 3" x 3" x 3/8"
2 L's 4" x 3" x $\frac{1}{2}$"
2 Pls. 14" x 3/8"

Fig. 8.



SECTION $L_0\ U_2$

2 L's 3" x 3" x 3/8"
2 L's 4" x 3" x $\frac{1}{2}$"
2 Pls. 16" x $\frac{1}{2}$"
1 Cov. 18" x 3/8"

U_{10} U_{14} has no cover plate (Fig. 7) but there is an eccentricity due to the larger angles on the bottom flanges. The stress is -284,900 lbs., A = 21.22 sq. ins., e = 0.68 ins., I = 497.0 in.^{4}. The unit stress is 16,900 lbs. and efficiency 94.7%.

U_0 U_2 has no eccentricity (Fig. 5). The total stress is -21,100 lbs., r = 6.25 ins., and l/r is 38.4. The unit stress is 4440 lbs. and the allowable stress (Plate I) is 13,300 lbs. per sq. in. which gives an efficiency of 300%.

Art. 2. Lower Chords.

The investigation of the lower chords is simply dividing the stress by the area of the steel, and then dividing 16,000 by the result which gives the efficiency. Table III gives all necessary data in this investigation.

Art. 3. End Post.

The investigation of the End Post is the same as that for the Top Chord. As it is a deck bridge the wind stress is carried by the lateral system. The maximum stress is -364,100 lbs., area of section 33.48 sq. ins., eccentricity 1.15 ins.(Fig. 8), and moment of inertia 1063.7 in.^{4} The direct unit stress is 10,880 lbs., and bending 3870 lbs. making a total of 14,750 lbs. per sq. in., which gives an efficiency of 108.5%.

Art. 4. Vertical Web Members.

The vertical posts all have an equal maximum stress of -81,600 lbs. The least radius of gyration of the section

Fig.9.

Fig.10.

Fig.11.

Fig.12.

is 7.25 ins. (Fig. 13), l/r equals 66.3 and from Plate I the allowable stress is 11,350 lbs. per sq. in. The area of the section is 9.96 sq. ins., unit stress 8200 lbs. per sq. in., and efficiency 138.5%.

The sub-verticals (Fig. 14) are 20 ft. long and carry a stress of -42,100 lbs., r = 2.72 ins. and l/r = 95.2 The allowable stress is 9800 lbs., area 5.7 sq. ins., unit stress 7400 lbs., and the efficiency 133.0%.

The vertical post at L_0 carries the two panels U_0 U_2 and the total reaction of the four-panel truss adjacent. Fig. 16 shows the section. The stress is -139,200 lbs., the unit stress 7800 lbs. per sq. in. and the efficiency 145.0 %.

The columns at the center pier (Figs. 11 and 15) carry the total reaction of the truss and the wind stress amounting to -394,700 lbs. The radius of gyration is 12.42 ins., l/r equals 38.8, and the efficiency is 96%. Table III shows complete data.

Art. 5. Diagonal Web Members.

U_2 L_4, L_8 U_{10}, and L_{12} U_{14} are diagonal tension members, and the unit stress is merely the total stress divided by the area, and the results are tabulated in Table III.

The three compression members, L_4 U_6, U_6 L_8, and U_{10} L_{12} are of the same section (Fig. 20). The least radius of gyration is 5.27 ins., l/r = 64.7, the allowable stress is 11,470 lbs. per sq. in. The total stresses and efficiencies are shown in Table III.

SECTION L_4 U_4

4 L's 4" x 3" x 3/8"

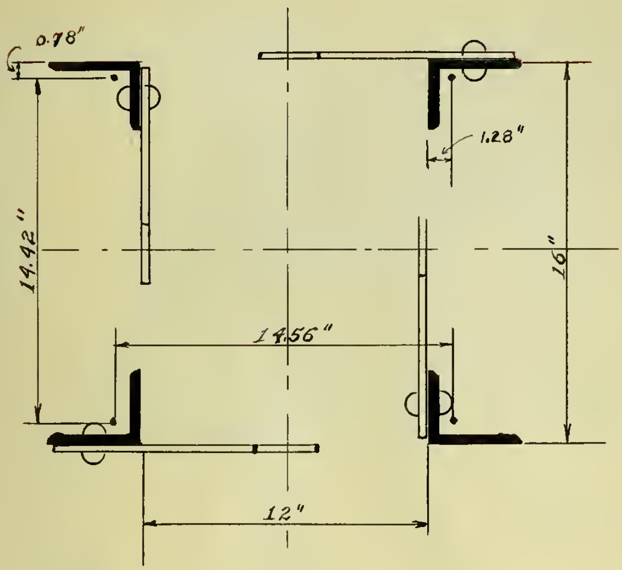


Fig. 13.

SECTION M_1 U_1

2- 7" Channels 9.75#

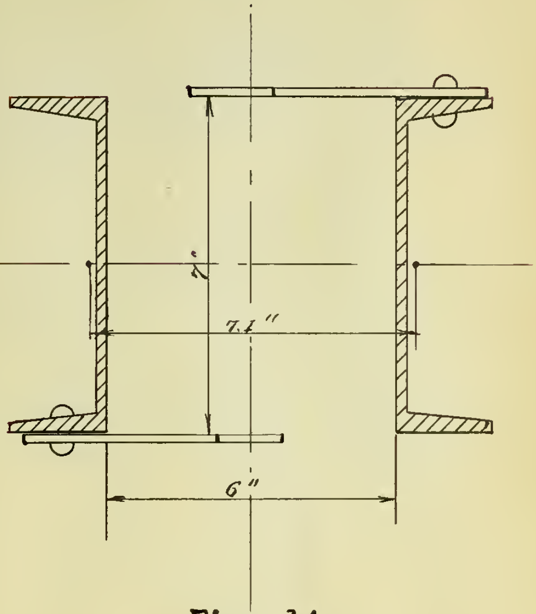


Fig. 14.

SECTION L_{14} U_{14}

4 L's 6"x 6" x 5/8"

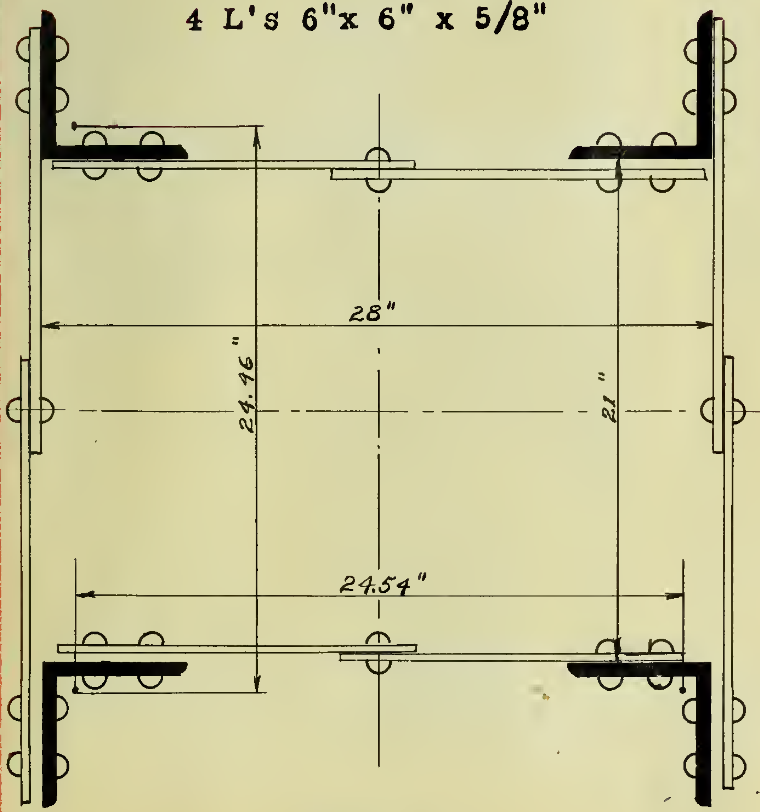


Fig. 15.

SECTION L_0 U_0

4 L's 5" x $3\frac{1}{2}$" x 9/16"

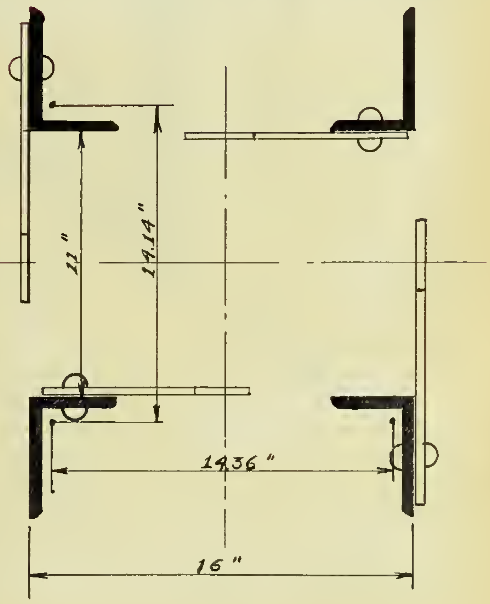


Fig. 16.

Fig. 17.

Fig. 18.

Fig. 19.

SECTION L_4 U_6

4 L's 5" x 3" x ½"

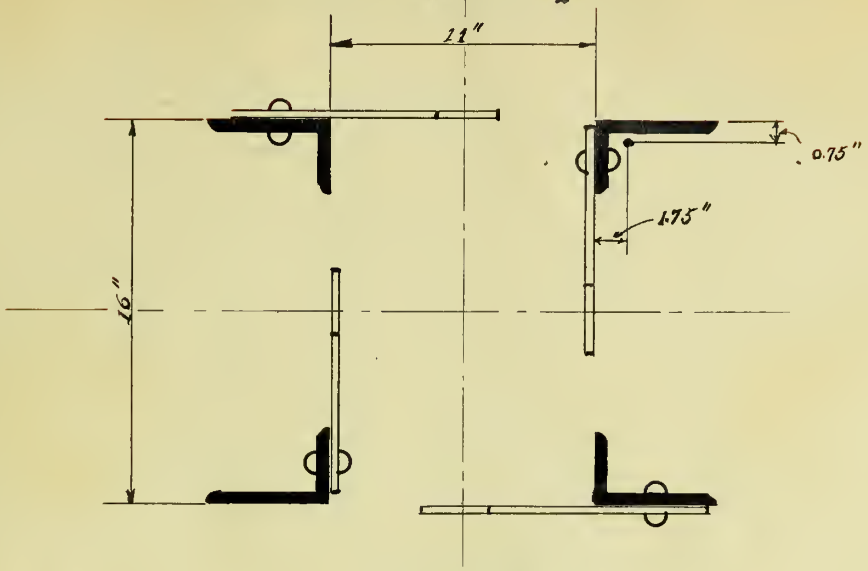


Fig. 20.

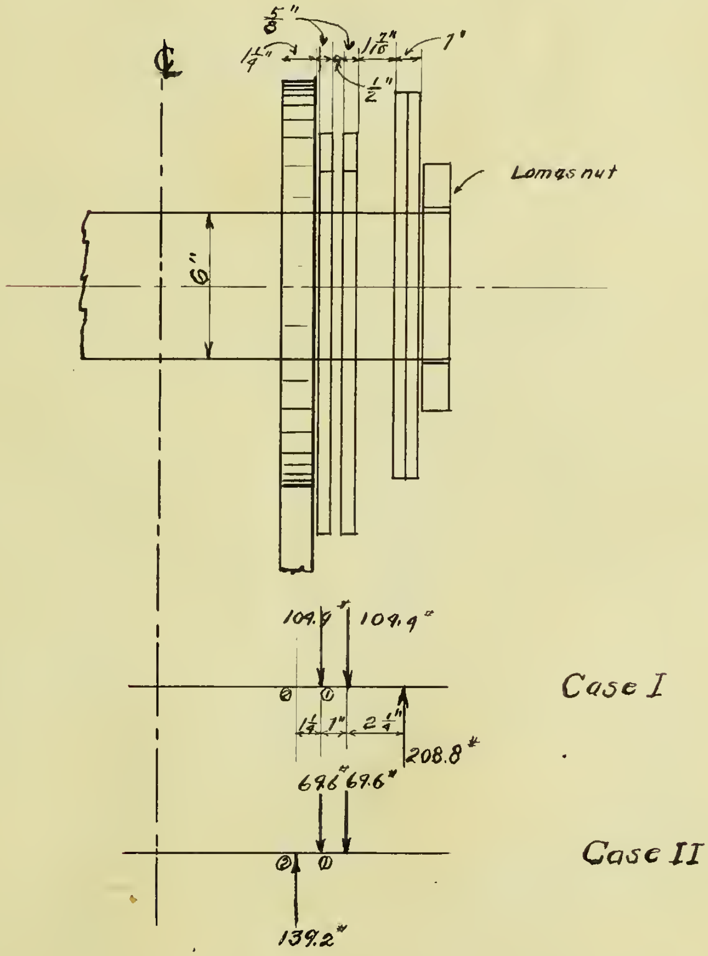


Fig. 21.

TABLE III.

MAIN TRUSS EFFICIENCIES.

Member	Maximum Stress	Section	Unit Stress	Per Cent Efficiency
L_0L_4	258,100	2 Eyes 5"x 1¼"	21,100	75.8
L_4L_8	521,200	4 Eyes 5" x 1¼"	20,820	76.8
L_8L_{12}	434,900	2 Eyes 5" x 2 1/8"	20,500	78.2
$L_{12}U_{14}$	403,300	2 Eyes 5" x 2"	20,160	79.4
U_2L_4	297,800	2 Eyes 5" x 1¼"	23,820	67.2
L_8U_{10}	194,700	2 Eyes 5" x 13/16"	23,900	67.0
U_0M_1	29,800	2 Bars 1" square	14,500	110.2
U_0U_2	-21,100	4 L's 2½"x 2½"x ¼"	-4,400	300.0
U_2U_6	-457,400	2 L's 3"x 3"x 3/8" 2 L's 4"x 3"x ½" 2 Pls. 14"x 3/8" 1 Cov. 18" x 3/8"	-22,550	74.4
U_6U_{10}	-545,500		-27,070	59.2
$U_{10}U_{14}$	-284,900	2 L's 3"x 3"x 3/8" 2 L's 4"x 3"x ½" 2 Pls. 14" x 3/8"	-16,900	94.7
L_4U_6	-155,400	4 L's 5"x 3"x ½"	-9,720	118.0
U_6L_8	-74,920	4 L's 5"x 3"x ½"	-4,550	252.0
$U_{10}L_{12}$	-254,500	4 L's 5"x 3"x ½"	-15,900	72.2
L_4U_4	-81,600	4 L's 4"x 3"x 3/8"	-8,200	138.5
M_1U_1	-42,100	2-7" Chans. 9.75#	-7,400	133.0
$L_{14}U_{14}$ wind	-304,700 -90,000 -394,700	4 L's 6"x6"x 5/8"	-13,850	96.0
L_0U_2	-364,100	2 L's 3"x 3"x 3/8" 2 L's 4"x 3"x ½" 2 Pls. 16" x ½" 1 Cov. 18" x3/8"	-14,700	108.5
L_0U_0	-42,100 -97,100* -139,200	4L's 5"x3½"x9/16"	-7,800	145.0

* Reaction from the four-panel truss.

Art. 6. Details.

Under this head, pins, pin-plates and rivets will be investigated. The pins at L_2, L_6, and L_{10} merely connect the eyebars and the bending moment is obtained directly. The shear, also, must be considered. At L_2, two eyebars 5" x1¼" carry 385,100 lbs. 179,050 lbs. on each bar, giving a moment of 223,550 in. lbs. on the pin. The pin is 5" in diameter. The formula:

$$S = \frac{Mc}{I}$$

for pins becomes

$$S = \frac{M}{0.098d^3}$$

From this the unit stress is 18,250 lbs. per sq. in., and the allowable being 24,000 lbs. per sq. in., the efficiency is 131.5%.

Similarly at L_{10}, two eyebars 5" x 2 1/8" carry 220,000 lbs. each, which gives a moment of 468,000 in. lbs. The resulting efficiency is 62.8%.

At L_6 there are four eyebars with a total stress of 511,000 lbs. The moment is 352,000 in. lbs., unit stress 28,700 lbs. per sq. in. and the efficiency is 83.6%. The efficiencies for shear can be obtained directly and are tabulated in Table IV. The bending efficiencies are also shown in this Table.

The pin at L_0 must be investigated for maximum moment both vertically and horizontally, the stress in the end post being resolved into vertical and horizontal components. In Fig. 21, Case I shows the vertical and Case II the horizontal.

Case I. Moment at (1) is 575,000 and at (2) 574,000 in. lbs.

Case II. " " " " 69,500 " " " 243,500 " "

The maximum resultant moment will be at (2) and is equal to

$$\sqrt{\overline{574,000}^2 + \overline{243,500}^2} = 624,000 \text{ inch pounds.}$$

The unit stress is 29,000 lbs. per sq. in. and the efficiency is 81.7%.

The two 5/8" pin-plates on each side of the end post (Fig. 9), have 8 - $\frac{3}{4}$" rivets in double shear and 6 in single shear, or 22 in single shear. One half of the load is 192,000 lbs. or 8740 lbs. on each rivet. The allowable stress on a $\frac{3}{4}$" rivet is 53.. lbs. which gives an efficiency of 60.7%. The bearing of the pin on the pedestal pin-plate is 208,000 divided by 6 or 34,880 lbs. per sq. in. giving an efficiency of 69.0%.

At L_8 the maximum moment is between the two eyebars of the lower chord and amounts to 220,000 in. lbs. The efficiency is 133.5%.

The remaining pins were computed in a similar manner and the complete results are tabulated in Table IV.

The pins at U_4, U_8, and U_{12} are the same and also M_1, M_5, M_7, and M_{11}.

TABLE IV.

DETAILS OF MAIN TRUSS - EFFICIENCIES.

Pins		Bending on Pin		Shear on Pin		Bearing on Pin-plates			Shear on Rivets		
	Diam. ins.	Unit Stress	Eff. %	Unit Stress	Eff. %	Member	Unit Stress	Eff. %	Member	Rivet Stress	Eff. %
L_0	6	29,400	81.7	7,400	162.2	plate	34,800	69.0	L_0U_2	8,740	60.7
L_4	5	37,800	63.5	6,560	183.0	"	25,400	95.0	diag.	5,180	102.0
L_8	5	18,000	133.5	11,000	108.8	"	15,100	159.0	vert.	5,100	104.0
L_{12}	5	38,300	62.8	11,000	108.8	"	29,600	81.2	diag.	8,470	62.6
L_2	5	18,250	131.5	6,560	183.0						
L_6	5	28,700	83.6	6,560	183.0						
L_{10}	5	38,300	62.8	11,000	108.8						
U_2	5	27,000	89.0	7,450	161.0	L_0U_2	28,900	83.0	L_0U_2	7,560	70.0
U_4	$3\frac{1}{2}$	9,500	252.0	4,250	273.0		22,900	105.0	L_4U_4	5,100	104.0
U_{10}	5	28,200	85.2	6,440	187.0	$U_{10}U_{14}$	26,900	89.3	U_6U_{10}	6,300	84.0
U_{14}	6	29,900	80.4	5,740	209.0	$L_{14}U_{14}$	40,600	59.0		5,800	91.5
M_1	$2\frac{1}{2}$	9,450	252.0	4,280	280.0		22,400	107.0			
M_3	5	15,200	157.0	7,600	158.0						
M_9	5	6,450	372.0	4,960	242.0						
M_{13}	5	33,000	72.8	10,300	116.0						

PART III.

INVESTIGATION OF THE SMALL TRUSSES.

There are six small spans of which four are 3-panel and two are 4-panel trusses. The photograph (Fig. 17) shows the 4-panel truss and its supports. The panel lengths and widths of trusses are the same as the main span. The 4-panel truss is 16 ft. deep and the 3-panel truss is 12 ft. Table V is a summary of the weights of the members, and this total was used in the computation of stresses, as in the main trusses. The weights of the details were assumed as 25% that of the main members. The live load for spans less than 100 ft. is 90 lbs. per sq. ft., which gives a live panel load of 29,700 lbs., and the dead panel load is 11,000 lbs.

Figs. 22 and 23 are sketches of the trusses, and Table VI is the stresses for both spans. These stresses were computed analytically and checked by graphics.

Figs. 24, 25 and 26 are cross-sections of the compression members of both trusses. The efficiencies of all members are shown in Table VII. The computations are similar to those of the main truss.

TABLE V.

WEIGHTS OF

THREE AND FOUR-PANEL TRUSSES.

Three-Panel Truss

No.	Member	Length	Wt. per ft.	Wt.	Total Wt.
2	Upper Ch.	60.0	30.0	1800	3600
8	Eyebars	25.0	8.93	220	1760
4	Eyebars	22.0	7.65	169	680
4	Bars	25.0	3.0	75	300
2	Struts	20.0	20.0	400	800
4	Bars	25.0	4.3	107	430
	Laterals				530
				Total	8100
		Four-Panel Truss			
2	Upper Ch.	80.0	38.0	3040	6080
8	Eyebars	22.0	7.65	170	1360
8	Eyebars	26.0	10.2	265	2120
6	Posts	16.0	38,0	608	3650
8	Eyebars	26.0	5.95	155	1240
4	Counters	26.0	2.6	68	270
3	Struts	20.0	20.0	400	1200
	Laterals etc.				880
				Total	16,800

SKETCHES

OF SMALLER TRUSSES.

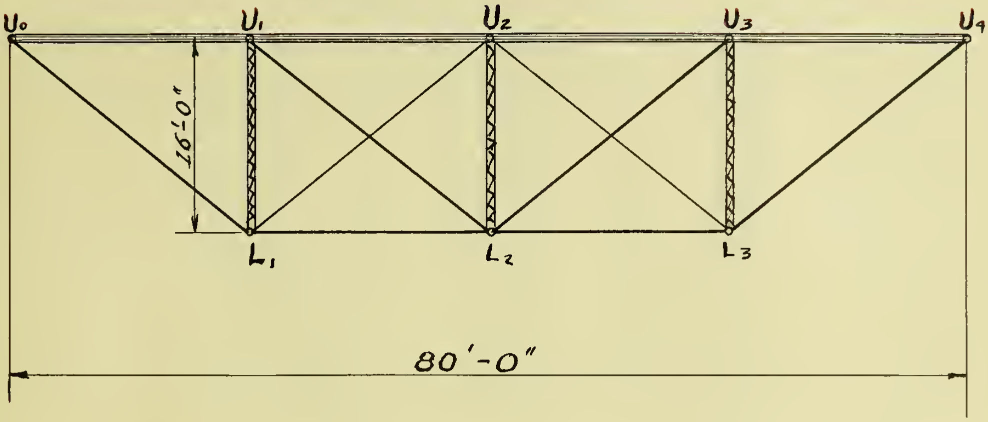


Fig. 22.

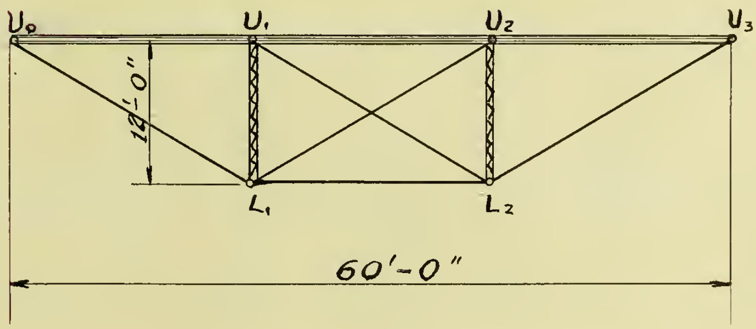


Fig. 23.

TABLE VI.

STRESSES

SMALL SPAN TRUSSES.

Four-Panel Truss

Member	Dead Load	Live Load	Impact	Total
U_0U_1	-20,600	-55,700	-14,700	-91,000
U_1U_2	-27,500	-74,200	-19,500	-121,200
U_0L_1	26,400	71,400	18,800	116,600
L_1U_1	-16,500	-44,500	-11,700	-72,700
L_1L_2	20,600	55,700	14,700	91,000
L_2U_2	-11,000	-29,700	-7,800	-48,500
L_1U_2	0	8,750	2,300	11,050
U_1L_2	8,800	35,600	9,400	53,800
	Three-Panel Truss.			
U_0U_1	-17,100	-49,500	-13,700	-80,300
U_1U_2	"	"	"	"
L_1L_2	"	"	"	"
U_0L_1	19,800	57,400	15,900	93,100
L_1U_1	-10,250	-27,600	-7,700	-45,550
U_1L_2	0	18,300	5,100	23,400

SECTION UPPER CHORD

FOUR-PANEL TRUSS.

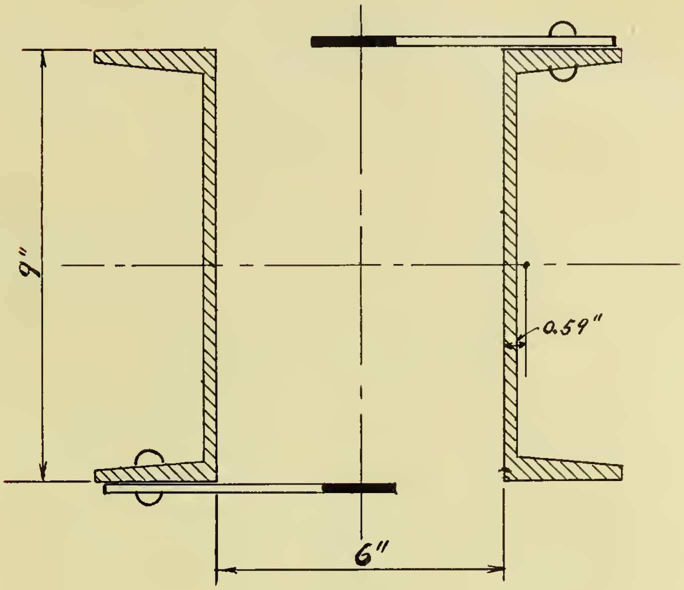


Fig. 24.

THREE-PANEL TRUSS

SECTION U_0U_2

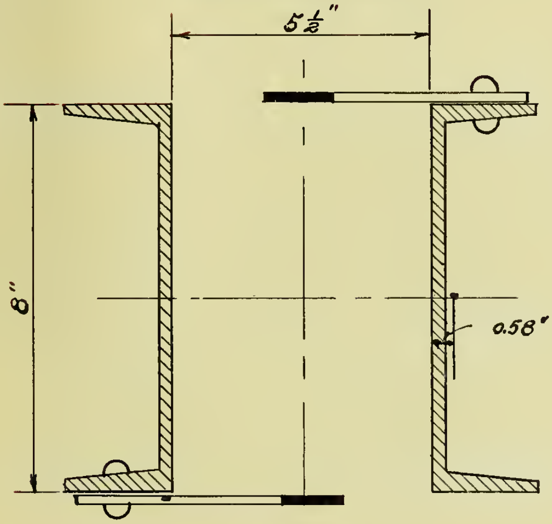


Fig. 25.

SECTION L_1U_1

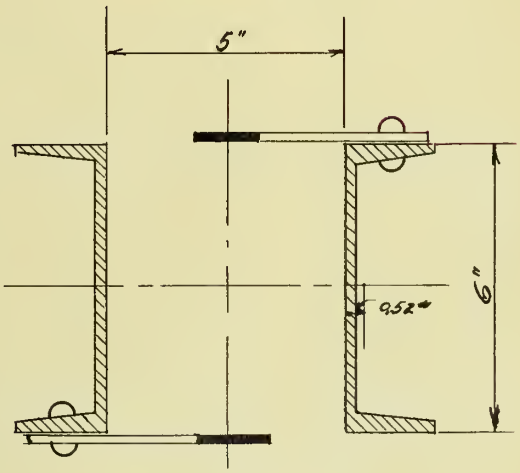


Fig. 26.

TABLE VII.

EFFICIENCIES.

OF SMALL TRUSSES.

Member	Stress	Section	Unit Stress	Per Cent Efficiency
	Four-Panel Truss			
U_0U_1	- 91,000	2-9" Chs. 15#	-10,300	106.5
U_1U_2	-121,000	2-9" Chs. 15#	-13,700	80.3
U_0L_1	116,600	2 Eyes 3"x 1"	19,400	82.4
L_1L_2	91,000	2 Eyes 3"x ¾"	20,200	79.2
L_1U_1	-72,700	2-9" Chs. 15#	-8,250	145.5
U_1L_2	53,800	2 Eyes 2"x7/8"	15,400	104.0
L_1U_2	11,050	1 Bar 7/8" sq.	12,600	127.3
L_2U_2	-48,500	2-9" Chs. 15#	-5,500	218.0
	Three-Panel Truss			
U_0U_1	-80,300	2-8" Chs.11¼#	-12,000	89.5
U_1U_2	-80,300	2-8" Chs.11¼#	-12,000	89.5
L_1L_2	-80,300	2 Eyes 3"x ¾"	17,850	89.8
U_1L_2	23,400	2 15/16" sq.	12,450	128.5
U_0L_1	93,100	2 Eyes 3"x7/8"	17,750	90.2
L_1U_1	-45,550	2-6" Chs. 8#	-9,600	121.0

PART IV.

INVESTIGATION OF TOWERS.

There are four towers, all of different heights, the tallest being 60 ft. This one carries the greatest load as it supports one end of a 3-panel and one end of a 4-panel truss. A general sketch of the tower is shown in Fig. 27. The photograph (Fig. 17) shows the upper half of the tower at the left.

In computing the live, dead and wind load stresses there are two conditions to be considered; structure loaded and structure unloaded. For the former the wind is 30 lbs. per sq. ft. on the exposed surfaces of all trusses and the floor as seen in elevation in addition to a uniform load of 150 lbs. per lin. ft. of structure applied on the loaded chord, and for the latter 50 lbs. per sq. ft. on the exposed surface of the truss and floor as seen in elevation.

The total reaction from the 4-panel truss is 97,000 lbs. and from the 3-panel truss is 64,000 lbs. As it is evident that the maximum stresses will come under the greater load only that side will be investigated.

The stresses were computed from the following formulae:

For dead and live stresses.

$$S_{ab} = -P \sec \theta$$

$$S_{aa'} = -P \tan \theta \cos B$$

$$S_{bc} = -(P + W) \sec \theta$$

$$S_{bb'} = -W \tan \theta \cos B$$

For wind stresses.

$$S_{ab} = -H \frac{h + h_1}{b_1} \sec \theta$$

$$S_{ba'} = H\left(\frac{h + h}{b_1} - \frac{h}{b}\right) \sec \theta_1$$

$$S_{bc} = -H \frac{h + h_1 \quad h_2}{b_2} \sec \theta$$

$$S_{bb'} = -H + 2H \frac{h + h}{b_1} \tan \theta$$

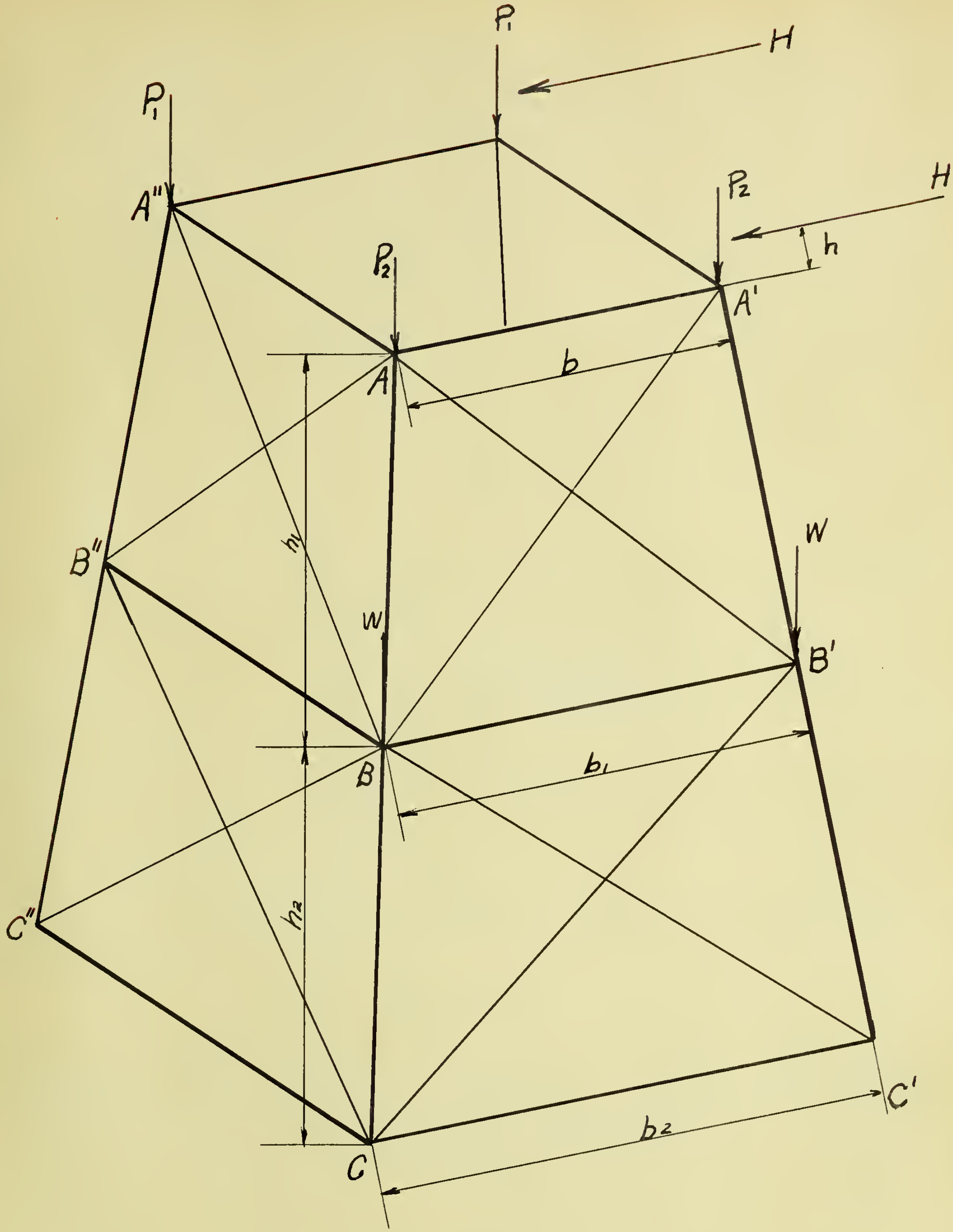


Fig. 27.

SKETCH OF TOWER.

P = direct load

θ = angle AB makes with the vertical

B = angle of horizontal projection of AC with CC'.

W = load due to the weight applied at B

H = wind load

h = height of H above AA'

h_1 = vertical distance AB

h_2 = vertical distance BC

b = length AA'

b_1 = " BB'

b_2 = " CC'

For the structure loaded H equals 27,900 lbs. and unloaded 26,000 lbs. Sec θ = 1.012, tan θ = 0.236, h = 3 ft., $h_1 = h_2$ = 30 ft., b = 20 ft., b_1 = 30 ft. b_2 = 40 ft., sec θ_1 = 1.20, and B = 45°.

The greatest stresses are evidently produced with the structure loaded.

Table VIII shows the maximum stresses, sections, and efficiencies.

TABLE VIII.

Members	Total Stress	Section	Unit Stress	Per Cent Efficiency
AB	-122,600	4 L's 5"x 3"x$\frac{3}{8}$"	-10,700	116.5
BC	-150,500	"	-13.150	95.0
AA'	-31,800	4 L's $3\frac{1}{2}$"x3"x$\frac{5}{16}$"	-4,100	217.0
AB'	35,700	2 Bars 1 1/8"	17,850	90.0
B'C	20,800	2 Bars $\frac{3}{4}$"	13,850	115.3
BB'	-15,400	4 L's$2\frac{1}{2}$"x$2\frac{1}{2}$"x$\frac{1}{4}$"	3,630	325.0

PART V.

FOUNDATIONS.

The time occupied in this thesis is too limited to make a long and exhaustive investigation of the foundations as that would involve a careful study of the conditions of the soil or rock, and the quality of the concrete or masonry used, which is beyond the purpose of this investigation. All that will be attempted here is a note of careful inspection.

The abutment at the north end of the bridge which is of masonry, shows signs of weakness at the top due to the disintegration of the stone, which is crumbling off to some extent.

The center and north piers of the main spans which go down to rock seem to be in very good condition. The center pier carries a total load of 1,208,800 lbs. or approximately 20 tons per sq. ft. on the foundation which is allowable.

The pier at the south end of the span has recently been rebuilt of concrete, and put down to rock (Fig. 9).

The abutment at the south end of the bridge appears to be in fair condition.

PART VI.

PHYSICAL CONDITIONS.

The steel of the main members of the trusses was found to be in fair condition except that the whole structure is badly in need of paint. The bridge has probably not been painted for several years, and what paint is left is rapidly peeling off. No extensive damage from corrosion is evident on the main truss members.

The most notable effects of corrosion were upon angles of the lateral struts at the base of the towers and especially one between the two L_0's of the main truss on the south end of the span. This strut is composed of four latticed angles and as it is inclined parallel to the end post one angle is in a position to hold water. This angle has almost disappeared, a shell being left that can be crumbled with the fingers. The arrow Fig. 9 shows the location of this angle. The other struts were not in such a bad condition.

The floor beams are in very good condition which is due no doubt to the fact that they are protected by the floor. The paint of the floor beams is in fair condition.

As much as was examined of the floor stringers seemed to be in very good shape and the flooring itself, as far as could be determined, was good.

PART VII.

CONCLUSIONS.

In taking up the conclusions of this investigation all that will be attempted will be merely to point out where failure might occur. The weakest condition found was that of the wooden floor stringers which gave an efficiency in bending of only 38.2% under the concentrated load. This gives unsafe results especially as the timbers are old and their exact condition unknown. The loading taken is not usual on this bridge,but could be possible. The steel floor beams give a slightly higher efficiency of 43.0% which shows consistency in design.

The chords of the main truss average about 75.0% efficiency and vary only about 10%, except U_6 U_{10} which drops to 59.2%, it being the lowest. This is very low and any defect or corrosion of the material would make it dangerous with this load.

The details show low efficiencies. At L_{10}, 2 - 5"x $2\frac{1}{8}$" eyebars on a 5" pin gives only 62.8% while the members show 78.2% which indicated a lack of consistency. Thinner eyebars would have increased the efficiency of the pin.

At L_0 the bearing of the pin on the pedestal pin-plate gives 69.0% efficiency and at U_{14}, 59.0%. A careful study of Tables III and IV will show that the efficiencies of the details are lower than those of the main span members.

The small trusses have higher efficiencies, being above 80%.

The towers are safe, having efficiencies above 90%.

It is evident, then, that for a very heavy concentrated load the floor might be expected to break through or a floor beam fail. Of course in reality, if there was any doubt about the safety of the floor, reinforcing plank would be put down for the roller or traction engine and the whole conditions would be changed.

For the main trusses a live load of 70 lbs. per sq. ft. would stress some of the details close to the elastic limit and hence would not be safe. This load could be possible with a crowd of people but is not probable.

END.